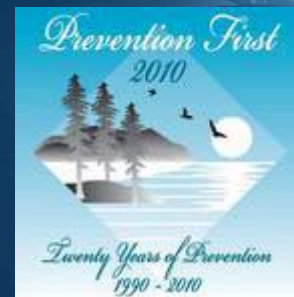


New ASCE Standard for Seismic Design of Piers and Wharves

Presented by:

Gayle Johnson
Halcrow, Inc.
Oakland, CA

October 20, 2010



Halcrow

Agenda

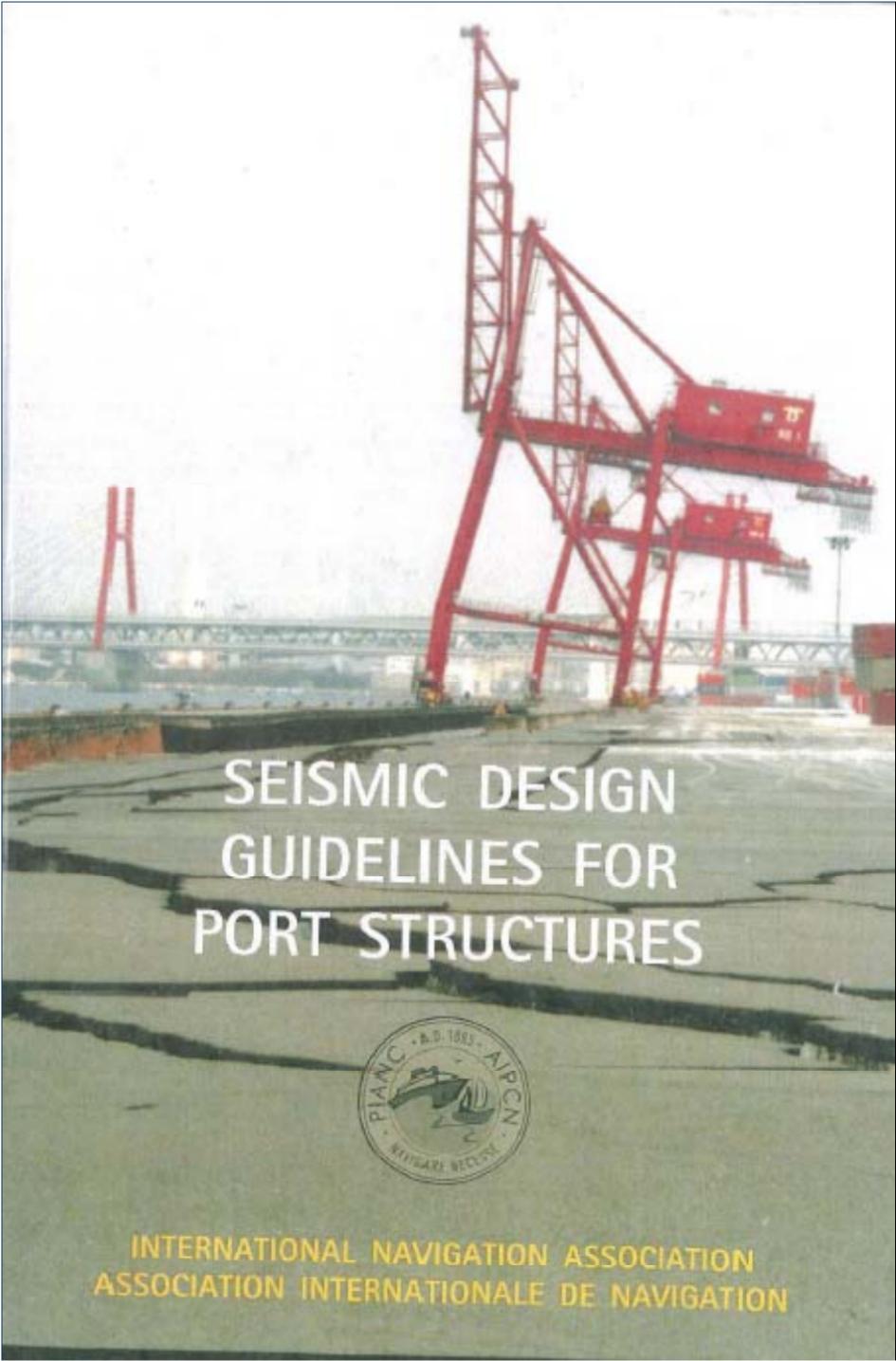
- Background / history of standards
- Why we are where we are
- What's coming
- Potential impact to the marine industry

ASCE Standards Committee

- Formed in 2005
- National committee of > 40 professionals
- Owners, consultants, and academics
- Geographically diverse
- Heavy geotechnical emphasis
- Includes loading and design details specific to marine structures
- Funding by US Navy
- Our tribe: “We’re great” Building guys suck.

What will these new standards do?

- Codify current practice of performance-based seismic design
 - National consensus document
- Build on work done by others specifically for the marine industry
 - California State Land Commission (MOTEMS)
 - Port of Los Angeles
 - Port of Long Beach
 - PIANC



SEISMIC DESIGN
GUIDELINES FOR
PORT STRUCTURES



INTERNATIONAL NAVIGATION ASSOCIATION
ASSOCIATION INTERNATIONALE DE NAVIGATION

MARINE OIL TERMINAL
ENGINEERING AND MAINTENANCE
STANDARDS



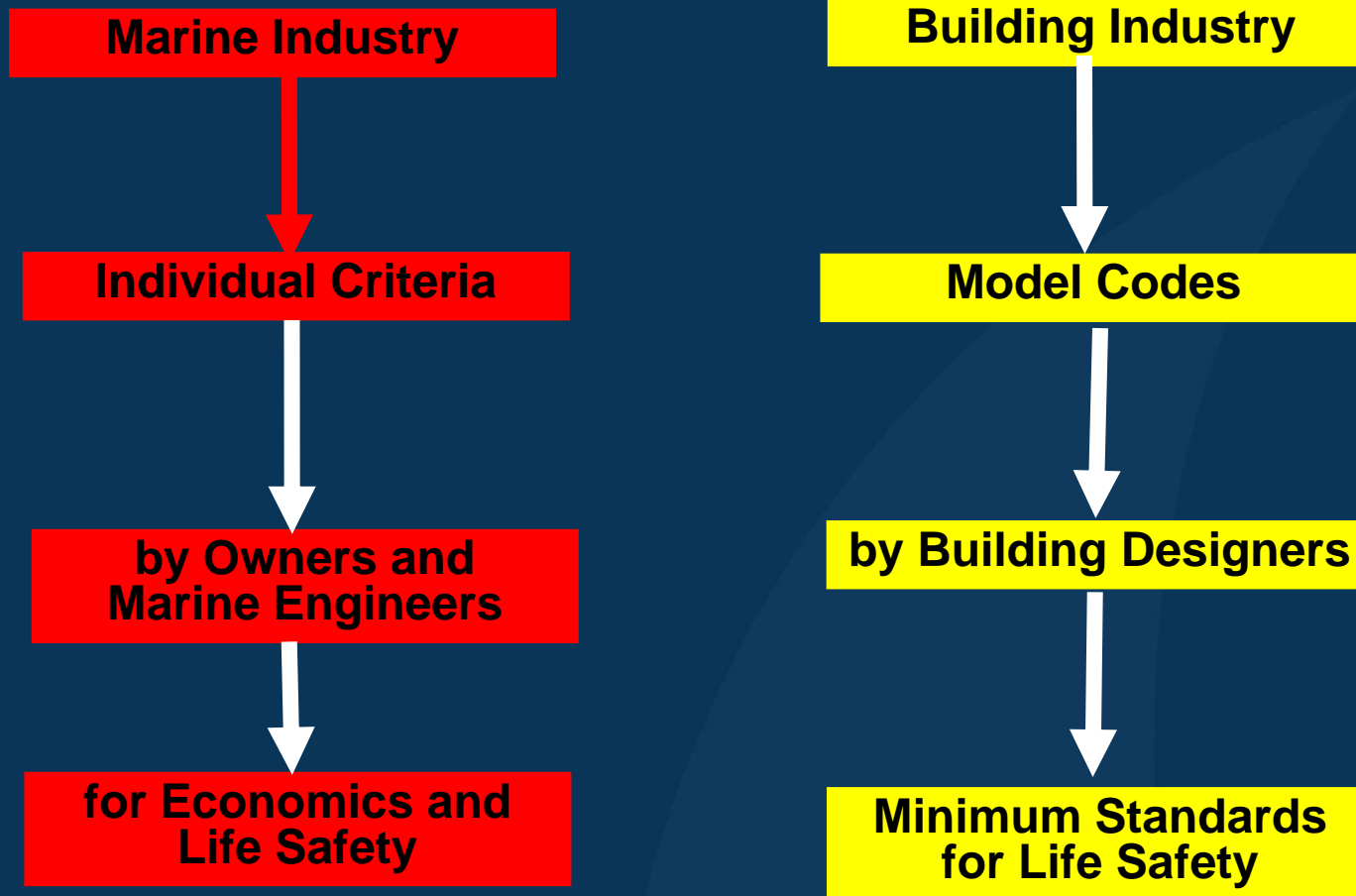
CALIFORNIA STATE LANDS COMMISSION
MARINE FACILITIES DIVISION

May 2002

Why is this necessary ?

- Billions of dollars of construction in seismic regions
 - Performance-based design being used routinely on a project basis
- Existing marine codes have limited standing
- Conventional building codes still often take precedence
 - Enforcement by local building officials
- Conventional code development controlled by building designers
 - Major changes to those codes

Code Development – 2 separate paths



Marine Industry Historic Practice

- Through 1980's equivalent lateral force methods – mostly AASHTO based
- Lateral force often specified, not calculated for each project using R values, site factors, etc.
- Each major port (POLA, POLB, POAK) set their own criteria
 - POLA – 1981 used $V = 0.12 W$

| Economics | | |
|---------------------|------------------|-------------|
| BERTHS 216-218 POLA | JOB No. 7052-5-2 | SHEET No. 1 |
| DESIGN CRITERIA | DESIGNED BY | DATE MAY 51 |
| | APPROVED | |

B. SEISMIC:

PSEUDO STATIC FORCE OF 0.12 g BOTH TRANSVERSE AND LONGITUDINAL DIRECTION.

g = TOTAL DEAD LOAD OF WHARF + CRANE DEAD LOAD.

ASSUMED CRANE DEAD LOAD = 1000 K

Performance-based design

- PSHA's common by mid-1980s
- Two level design
- Port of Oakland Example
- L1: **20% in 50 years (240 year RP)**
 - 7% damping
 - 0.35g PGA
 - 0.95g Spectral peak
 - Divide spectral peak by “risk factor” of 8
- L2: **50% in 50 years (72 year RP)**
 - 5% damping
 - 0.25g PGA
 - 0.65g Spectral peak
 - Use “risk factor” of 4 (ductility before spalling)
- Smaller L2 earthquake governs design (**$V = 0.16 W$**)

1994 Port of Oakland Design

SEISMIC CRITERIA:

| | 1 | 2 |
|---|---------|----------|
| EARTHQUAKE LEVEL | EXTREME | MODERATE |
| TYPE OF SEISMIC EVENT | | |
| PROBABILITY OF EXCEEDANCE IN 50 YEARS | 20% | 50% |
| PEAK GROUND ACCELERATION | 0.35g | 0.25g |
| % DAMPING | 5 | 5 |
| DUCTILITY/RISK FACTOR, Z | 5 | 2 |
| PEAK OF DAMPED SPECTRAL ACCELERATION (PDSA) | 0.95 W | 0.65 W |

$$V_u = \frac{PDSA}{Z} W$$

1999 Port of Oakland Design

SEISMIC CRITERIA:

| | | | |
|---|-------|-------|-------|
| EARTHQUAKE LEVEL | 1 | 2 | 3 |
| PROBABILITY OF EXCEEDANCE IN 50 YEARS | 50% | 20% | 10% |
| % DAMPING | 5% | 5% | 5% |
| TOP OF PILE FORCE REDUCTION FACTOR, R | 2 | 4 | 5 |
| IN-GROUND PILE FORCE REDUCTION FACTOR, R | 1 | 2 | 2 |
| PEAK GROUND ACCELERATION (PGA) | 0.25g | 0.37g | 0.44g |
| PEAK OF DAMPED SPECTRAL ACCELERATION (PDSA) | 0.82g | 1.16g | 1.38g |

NOTE: PGA AND PDSA REPRESENT GROUND MOTION 10 FEET BELOW SURFACE FOR THE COSM CONFIGURATION.

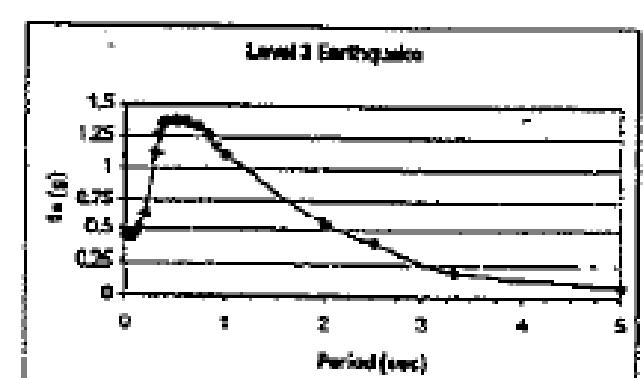
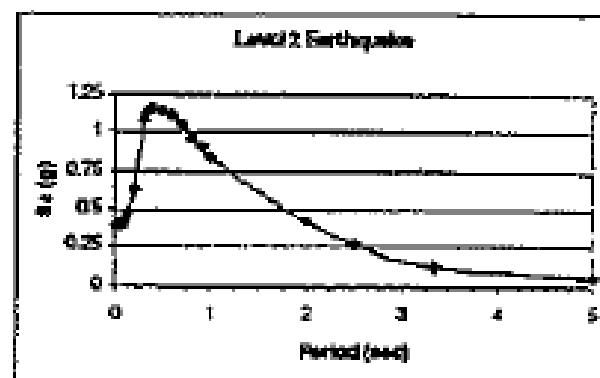
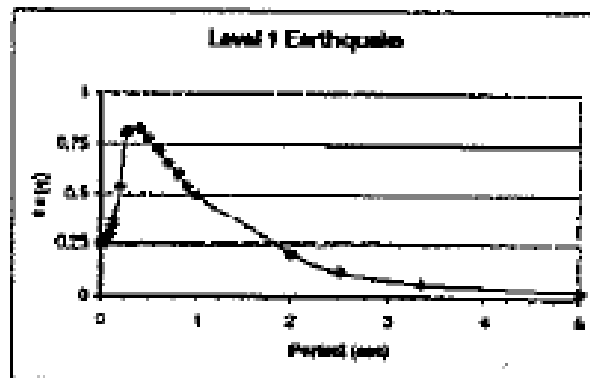
SEISMIC LOAD COMBINATIONS

- (1) 1.4 DL + 1.0 EQ + 0.1 (VERTICAL LIVE LOAD FOR PILE DESIGN)
- (2) 0.9 DL + 1.0 EQ

SITE-SPECIFIC RESPONSE SPECTRA HAVE BEEN DEVELOPED FOR THIS PROJECT BY SUBSURFACE CONSULTANTS, INC. MARCH 1998. FROM THESE SITE-SPECIFIC RESPONSE SPECTRA, THE FOLLOWING DESIGN RESPONSE SPECTRAS WERE DEVELOPED:

SLOPE DEFORMATION CRITERIA

| SEISMIC EVENT | DEFORMATION LIMITS |
|---------------|--------------------|
| POST-LEVEL 1 | MINIMAL |
| POST-LEVEL 2 | LESS THAN 6" |
| POST-LEVEL 3 | LESS THAN 12" |



2003 Port of Oakland Design

1. SEISMIC CRITERIA:

EARTHQUAKE LEVEL
 PROBABILITY OF EXCEEDANCE IN 50 YEARS
 % DAMPING
 RESPONSE MODIFICATION FORCE REDUCTION FACTOR, R:
 ABOVE GROUND PILE FORCE
 IN-GROUND PILE FORCE
 PEAK GROUND ACCELERATION (PGA)
 PEAK OF DAMPED SPECTRAL ACCELERATION (PDSEA)

| | | |
|-------|-------|-------|
| 1 | 2 | 3 |
| 50% | 20% | 10% |
| 5% | 5% | 5% |
| 2 | 4 | 5 |
| 1 | 2 | 2 |
| 0.37g | 0.46g | 0.50g |
| 0.85g | 1.20g | 1.40g |

SEISMIC LOAD COMBINATIONS

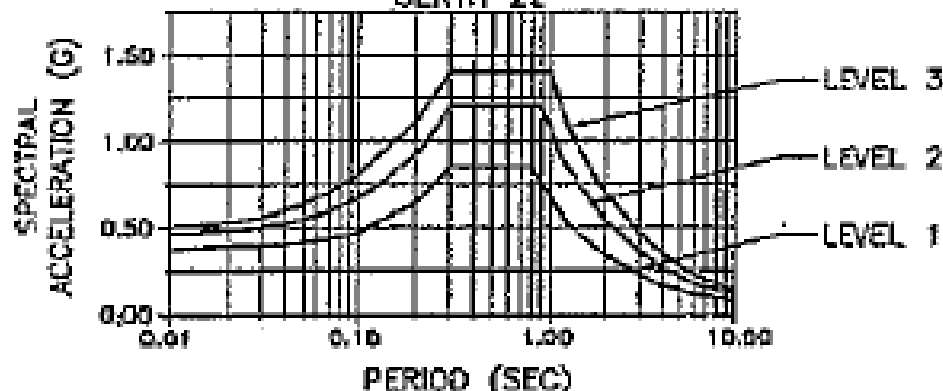
- (1) 1.4 DL + 1.0 EQ + 0.1 (VERTICAL LIVE LOAD FOR PILE DESIGN)
- (2) 0.9 DL ± 1.0 EQ

SITE-SPECIFIC RESPONSE SPECTRA HAVE BEEN DEVELOPED FOR THIS PROJECT BY SUBSURFACE CONSULTANTS, INC. JUNE 2002. FROM THESE SITE-SPECIFIC RESPONSE SPECTRA, THE FOLLOWING DESIGN RESPONSE SPECTRAS WERE DEVELOPED:

SLOPE DEFORMATION CRITERIA

| SEISMIC EVENT | DEFORMATION LIMITS |
|---------------|--------------------|
| POST-LEVEL 1 | MINIMAL |
| POST-LEVEL 2 | LESS THAN 6" |
| POST-LEVEL 3 | LESS THAN 12" |

DESIGN RESPONSE SPECTRA
 BERTH 22



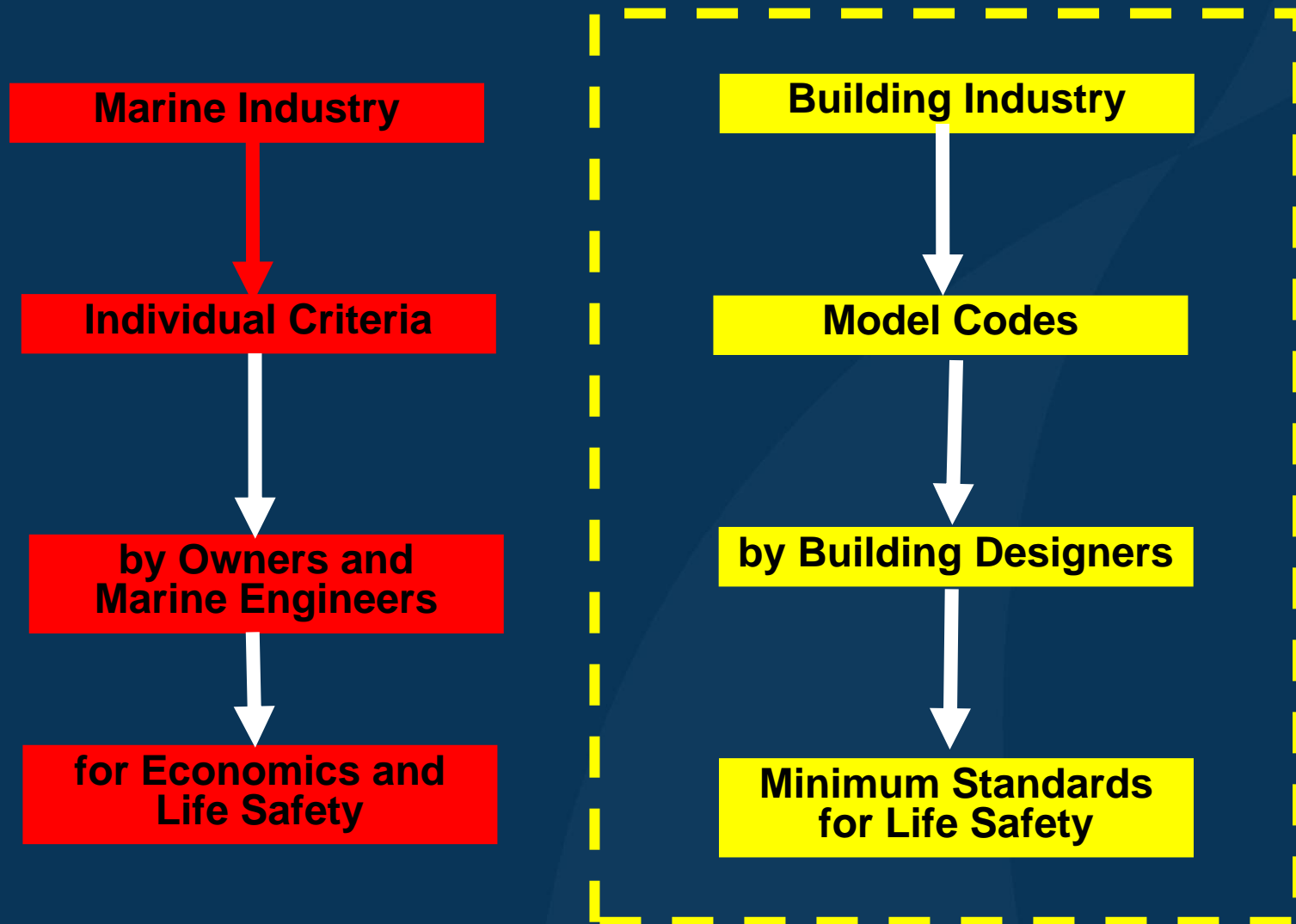
Were the low return periods unconservative ???

- Compare mid-80s Oakland design to a design using 1985 UBC
- $V = ZIKCSW$
 - Z = Zone factor (1.0 is highest for Zone 4)
 - I = Importance factor (1.0)
 - K = Factor for building type (0.67 for ductile moment resisting frame)
 - CS – Coefficients where CS need not exceed 0.14
- Express as ultimate load by applying a 1.4 factor
- $V = 0.13W$
- Less than the smaller L2 earthquake using the Port of Oakland internal criteria

Current marine industry seismic design practice

- Performance based design
 - Ports of Los Angeles, Long Beach, Oakland
 - MOTEMS for California Oil Terminals (State Law in 2006)
 - PIANC
- Two level earthquake
 - No damage level in small event
 - No collapse and repairable in large event
- Deformation based criteria
- Pushover analyses

Code Development – 2 separate paths



Meanwhile:

Building Codes – “The Early Days”

- Through 1997:
 - Three model building codes adopted by building officials in US
 - Note: Not all ports subject to local building official jurisdiction
 - Dominated by UBC / SEAOC “Blue Book”
 - “Nonbuilding structures” added in 1988
 - No specific reference to piers and wharves

“World domination” by building designers

- Post 1997:
 - Consolidation of 3 US Model Building Codes into IBC
 - FEMA Sponsored National Earthquake Hazards Reduction Program (NEHRP)
 - ASCE 7 becomes focal point
 - Different sponsoring organizations
 - Similar, but not identical, documents
 - Many of the same authors

Major changes to codes – not benign

- Some due to “lessons learned”, many changes for the sake of change
- Huge expansion of “nonbuilding structures”
 - Conflicts with existing industry practices and standards (not just piers and wharves)
- Major changes to ground motion definitions
 - Biggest effect outside of California

2000 NEHRP

14.6.6 Piers and Wharves:

14.6.6.1 General: Piers and wharves are *structures* located in waterfront areas that project into a body of water or parallel the shore line.

14.6.6.2 Design Basis: Piers and wharves shall be designed to comply with the *Provisions* and approved standards. *Seismic forces* on elements below the water level shall include the inertial force of the mass of the displaced water. The additional seismic mass equal to the mass of the displaced water shall be included as a lumped mass on the submerged element, and shall be added to the calculated *seismic forces* of the pier or wharf *structure*. Seismic dynamic forces from the soil shall be determined by the registered design professional.

The design shall account for the effects of liquefaction on piers and wharfs as required.

2003 NEHRP

- Task Committee of industry engineers
- Attempt to add performance-based design
- Crashed and burned

2003 NEHRP

14.3.3 Piers and wharves. Piers and wharves are structures located in waterfront areas that project into a body of water. Two categories of these structures are:

- a. Piers and wharves with general public occupancy, such as cruise ship terminals, retail or commercial offices, restaurants, fishing piers and other tourist attractions.
- b. Piers and wharves where occupancy by the general public is not a consideration and economic considerations (on a regional basis, or for the owner) are a major design consideration, such as container wharves, marine oil terminals, bulk terminals, etc., or other structures whose primary function is to moor vessels and barges.

These structures shall conform to the building or building-like structural requirements of the Provisions or other rational criteria and methods of design and analysis. Any methods used for design of these structures should recognize the unique importance of liquefaction and soil failure collapse mechanisms, as well as consider all applicable marine loading combinations, such as mooring, berthing, wave and current. Structural detailing shall be carefully considered for the marine environment.

14.3.3.1 Additional seismic mass. Seismic forces on elements below the water level shall include the inertial force of the mass of the displaced water. The additional seismic mass equal to the mass of the displaced water shall be included as a lumped mass on the submerged element, and shall be added to the calculated seismic forces of the pier or wharf structure.

14.3.3.2 Soil effects. Seismic dynamic forces from the soil shall be determined by the registered design professional. The design shall account for the effects of liquefaction on piers and wharves, as appropriate.

ASCE 7-05

15.5.6 Piers and Wharves.

15.5.6.1 General. Piers and wharves are structures located in waterfront areas that project into a body of water or parallel the shoreline.

15.5.6.2 Design Basis. In addition to the requirements of Section 15.5.1, piers and wharves that are accessible to the general public, such as cruise ship terminals and piers with retail or commercial offices or restaurants, shall be designed to comply with this standard.

The design shall account for the effects of liquefaction and soil failure collapse mechanisms, as well as consider all applicable marine loading combinations, such as mooring, berthing, wave, and current on piers and wharves as required. Structural detailing shall consider the effects of the marine environment.

Why was performance-based design rejected ?

- Two level performance criteria
- Levels of shaking / return periods viewed as “unconservative”
 - Consistent risk vs. life-safety
- Displacement based design not understood
- Inconsistency in building code geotechnical requirements not appreciated, not a big deal for buildings

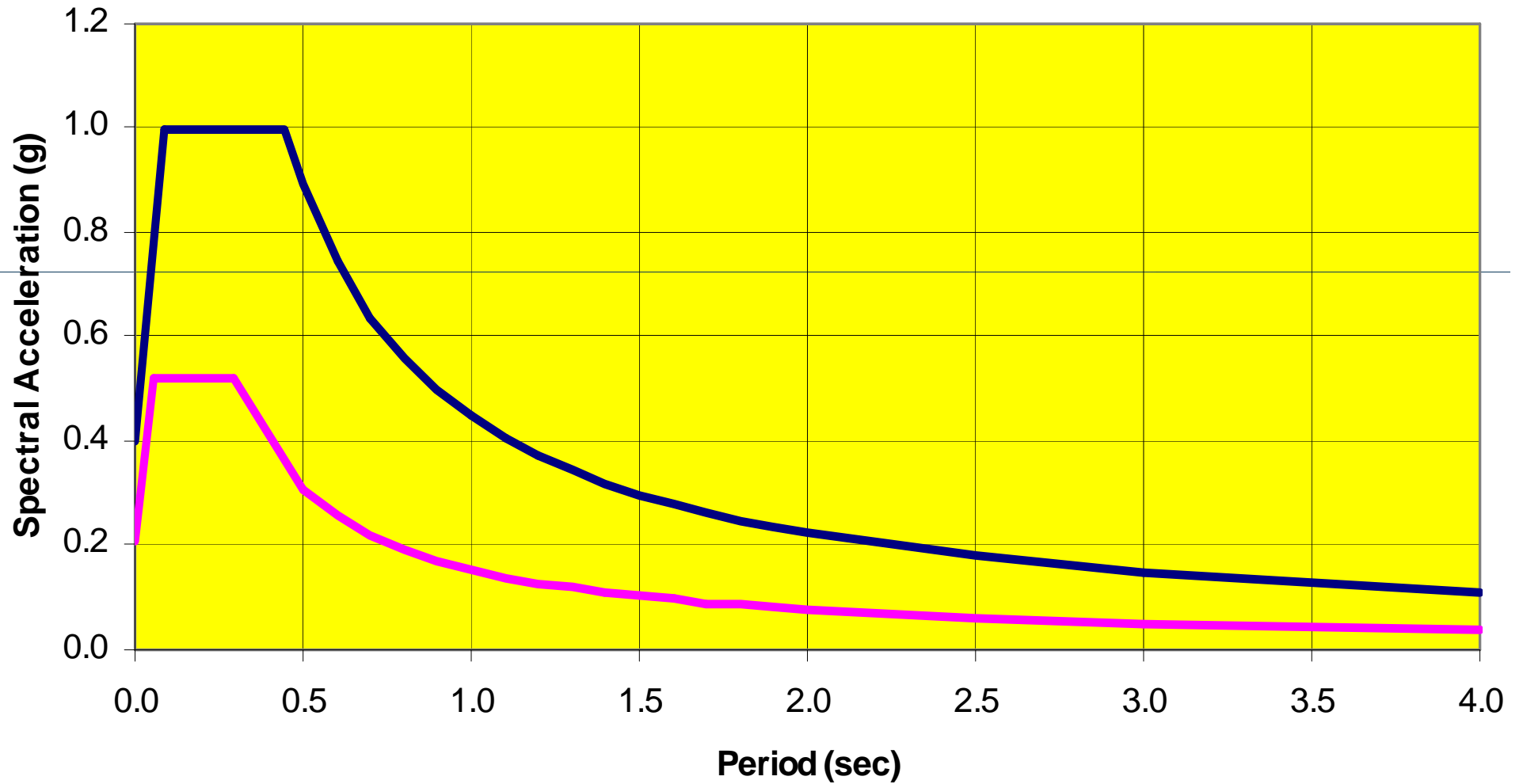
Performance Criteria

- ASCE 7 / IBC
 - Historically was single earthquake
 - 475 year return period
 - Life safety only
 - Now Maximum Considered Earthquake (MCE)
 - 2,500 year RP with deterministic cap
 - Allegedly for collapse prevention
 - Design Earthquake
 - 2/3 MCE
 - Life Safety
 - Really a single-level earthquake design for 2/3 MCE
 - Performance at higher level is presumed due to implied factors of safety for buildings

Why change the 475 year return period ?

- Increase ground shaking in Eastern US
 - 2% in 50 years
- Keep actual design values for California about the same
 - 2/3 factor
 - Justified by inherent 1.5 factor of safety

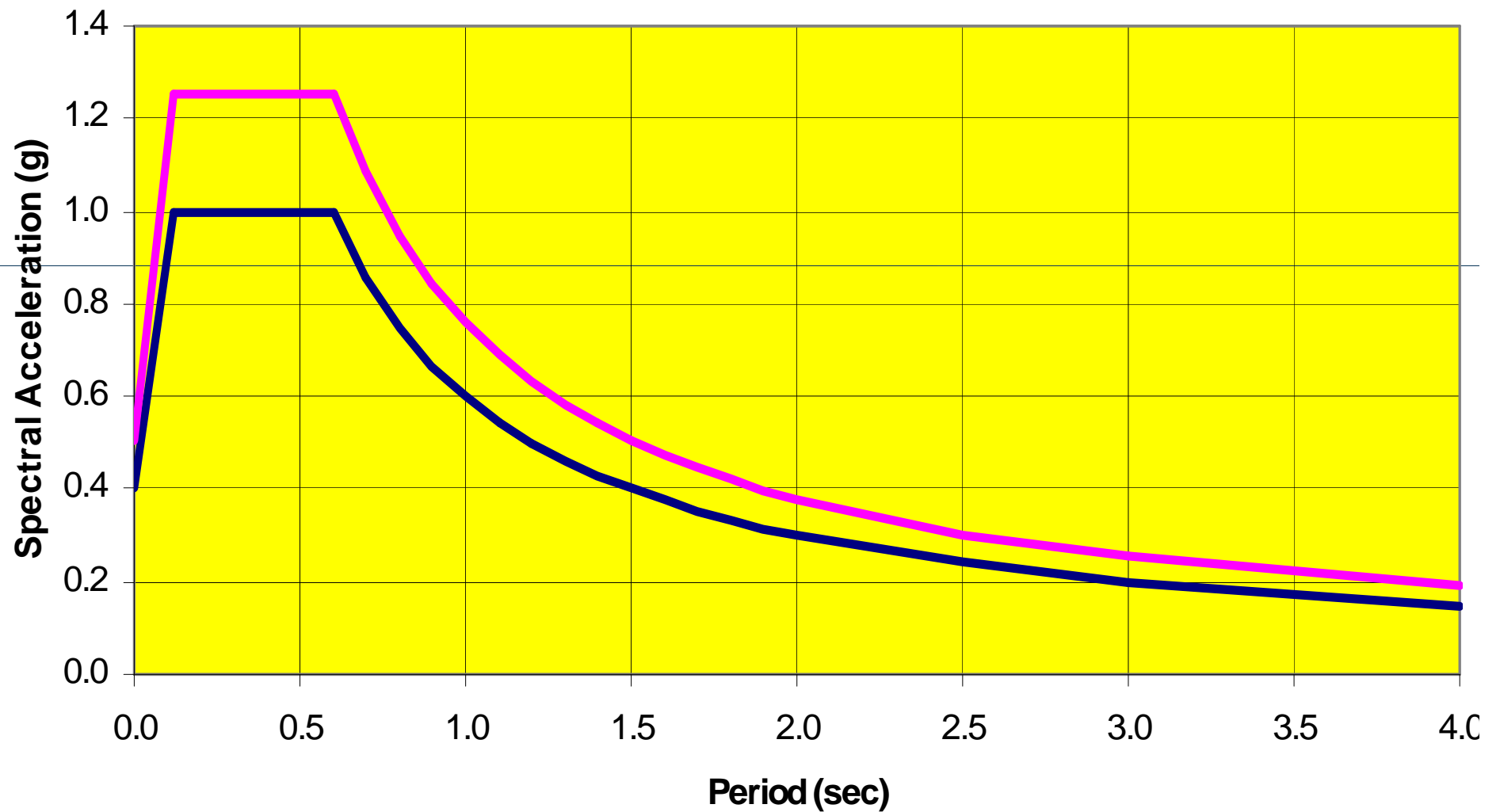
Charleston, SC Site Class D



— 2/3 NEHRP — USGS-10% in 50 Years

Oakland Outer Harbor Wharf, CA

Site Class D



— 2/3 NEHRP — USGS-10% in 50 Years

Port industry issues with changes

- Accelerations / forces can be scaled, displacements are not linear
- Massive ground failures occur in 2,500 year event that don't occur at 500 years
 - Can't just scale those events by $2/3$
- Hard to distinguish between damage states for life-safety and collapse prevention
 - Inherent 1.5 FS is only for buildings
 - Difference between life-safety and collapse is meaningless
 - Life safety hasn't been an issue
- Never have addressed lessons learned in ports

1989 Loma Prieta Earthquake



1989 Loma Prieta Earthquake



1995 Kobe Earthquake



1995 Kobe Earthquake



Halcrow

1995 Manzanillo, Mexico Earthquake



1995 Manzanillo, Mexico Earthquake



1999 Turkey Earthquake



1999 Turkey Earthquake



2004 Indonesia Earthquake / Tsunami



02/03/2005

New Standards

| Design Classification | Seismic Hazard Level and Performance Level | | | | | |
|-----------------------|--|-------------------|---|----------------------------------|------------------------|------------------------|
| | Operating Level Earthquake (OLE)* | | Contingency Level Earthquake (CLE)* | | Design Earthquake (DE) | |
| | Ground Motion Probability of Exceedance | Performance Level | Ground Motion Probability of Exceedance | Performance Level | Seismic Hazard Level | Performance Level |
| High | 50% in 50 years (72 year RP) | Minimal Damage | 10% in 50 years (475 year RP) | Controlled and Repairable Damage | as per ASCE-7 | Life Safety Protection |
| Moderate | n/a | n/a | 20% in 50 years (225 year RP) | Controlled and Repairable Damage | as per ASCE-7 | Life Safety Protection |
| Low | n/a | n/a | n/a | n/a | as per ASCE-7 | Life Safety Protection |

Does higher RP = more conservative ?

- ASCE 7
 - 2,500 year return period / non-collapse
 - 2/3 of that motion / life-safety
 - Lots of design coefficients e.g. “R” factor
- ASCE Piers and Wharves
 - Lower return periods
 - Controlled and repairable damage
 - “Failure” is more functional and economical
 - No real difference between life-safety and collapse

Table 3.3 CLE strain limits for "Controlled and repairable damage" per 2.4.2

| Section | Strain Limits | | |
|--|--|--|-------------------------------------|
| | Top of pile | In-ground | Deep in-ground (>10D _p) |
| Solid Concrete Piles - Doweled | $\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.025$ | $\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.008$ | $\varepsilon_c \leq 0.012$ |
| | $\varepsilon_s \leq 0.4\varepsilon_{smd} \leq 0.04$ | $\varepsilon_p \leq 0.025$ | $\varepsilon_p \leq 0.025$ |
| Hollow Concrete Piles ^a - Doweled | $\varepsilon_c \leq 0.006$ | $\varepsilon_c \leq 0.006$ | $\varepsilon_c \leq 0.006$ |
| | $\varepsilon_s \leq 0.6\varepsilon_{smd} \leq 0.06$ | $\varepsilon_p \leq 0.025$ | $\varepsilon_p \leq 0.025$ |
| Solid Concrete Piles - Fully Embedded | $\varepsilon_c = 0.005 + 1.1\rho_s \leq 0.025$ | $0.005 \leq \varepsilon_c \leq 0.008$ | $\varepsilon_c \leq 0.012$ |
| | $\varepsilon_p \leq 0.025, \varepsilon_s \leq 0.025$ | $\varepsilon_p \leq 0.025$ | $\varepsilon_p \leq 0.025$ |
| Steel Pipe Piles (concrete plug doweled connection) | $\varepsilon_c \leq 0.025$ | $\varepsilon_s \leq 0.025$ | |
| | $\varepsilon_s \leq 0.6\varepsilon_{smd} \leq 0.06$ | ~ | $\varepsilon_s \leq 0.035$ |
| Steel Pipe Piles (hollow steel section) - Fully Embedded | $\varepsilon_s \leq 0.025$ | $\varepsilon_s \leq 0.025$ | $\varepsilon_s \leq 0.035$ |
| | | | |
| Steel Pipe Piles (concrete filled) - Fully Embedded | $\varepsilon_s \leq 0.035$ | $\varepsilon_s \leq 0.035$ | |
| | | | |

POLA Experimental Program at UCSD
Current Experimental Work – Phase I



Final Semi-cycle from 7 to 8 in. Displacement
non-seismic Pile Test Unit

Tests at Oregon State University



Tests at University of Washington



Tests at University of Washington

1.75 % Drift



- 9% Drift

Halcrow

Advantages to industry specific standards

- Address common structural configurations
 - “Irregularities”
 - Sloping foundations
 - Battered piles
 - Strong beam / weak column
- Actual loading conditions
 - Kinematic
 - Mooring and berthing
- Code developers who work in the industry
 - Building designers think they know best
- Standing as “ASCE Mandatory Standard”
- Address issues like structural detailing

Table 7.1 Pile-to-deck connections

| Connection | Referenced Section | Permitted Moment Curvature Analysis Method(s) |
|--|---------------------------|--|
| Pipe Pile Connections | 7.4.2 | |
| Embedded Pile | 7.4.2.1 | Method B |
| Concrete Plug | 7.4.2.2 | Method B |
| Isolated Shell | 7.4.2.3 | Method B |
| Welded Dowels | 7.4.2.4 | NA |
| Welded Embed | 7.4.2.5 | Method B |
| Prestressed Concrete Pile Connections | 7.4.3 | |
| Pile Build-Up | 7.4.3.1 | Method A |
| Extended Strand | 7.4.3.2 | Method A |
| Embedded Pile | 7.4.3.3 | Method A |
| Dowelled | 7.4.3.4 | Method A or B |
| External Confinement | 7.4.3.5 | Method B |
| Hollow Dowelled | 7.4.3.6 | Method A or B |
| Other Connections ¹ | 7.4.4 | |
| Pinned Connection | 7.4.4.1 | Method B |
| Batter Pile | 7.4.4.2 | Method A or B |

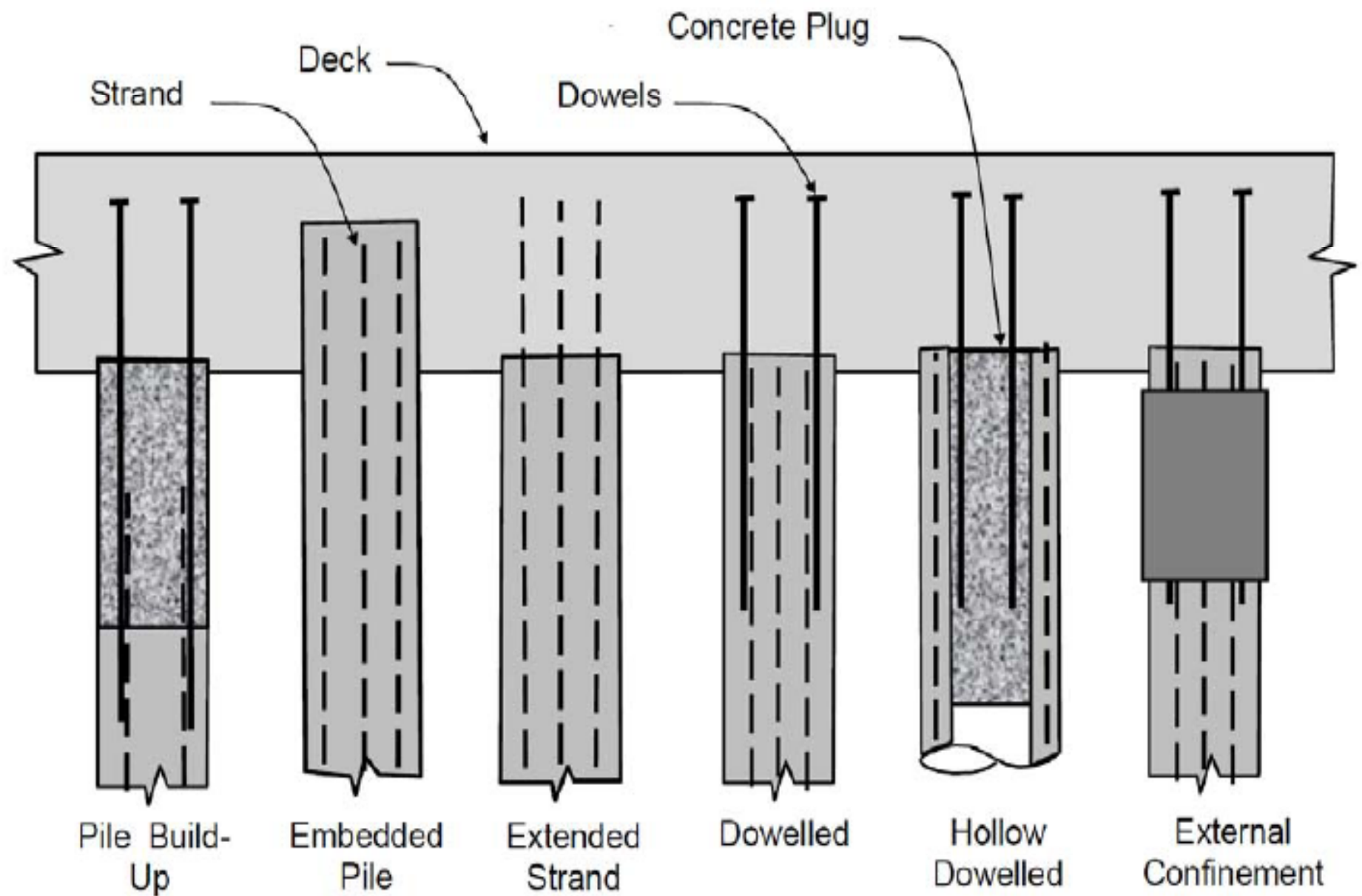


Figure C7.5 Examples of prestressed concrete pile connections

What's next ?

- Standard to be balloted this year
- Lengthy public review process
- Best case – published late 2011
- Over time – gain national standing and acceptance by building officials
- Continued application by marine industry

Acknowledgements

- Bob Harn – Berger ABAM
- Arul Arulmoli – Earth Mechanics
- Omar Jaradat – Moffatt & Nichol
- Nate Lemme – U.S. Navy

Questions ?

